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SEISMIC SOIL LIQUEFACTION AT THE WATERFRONT.(U)
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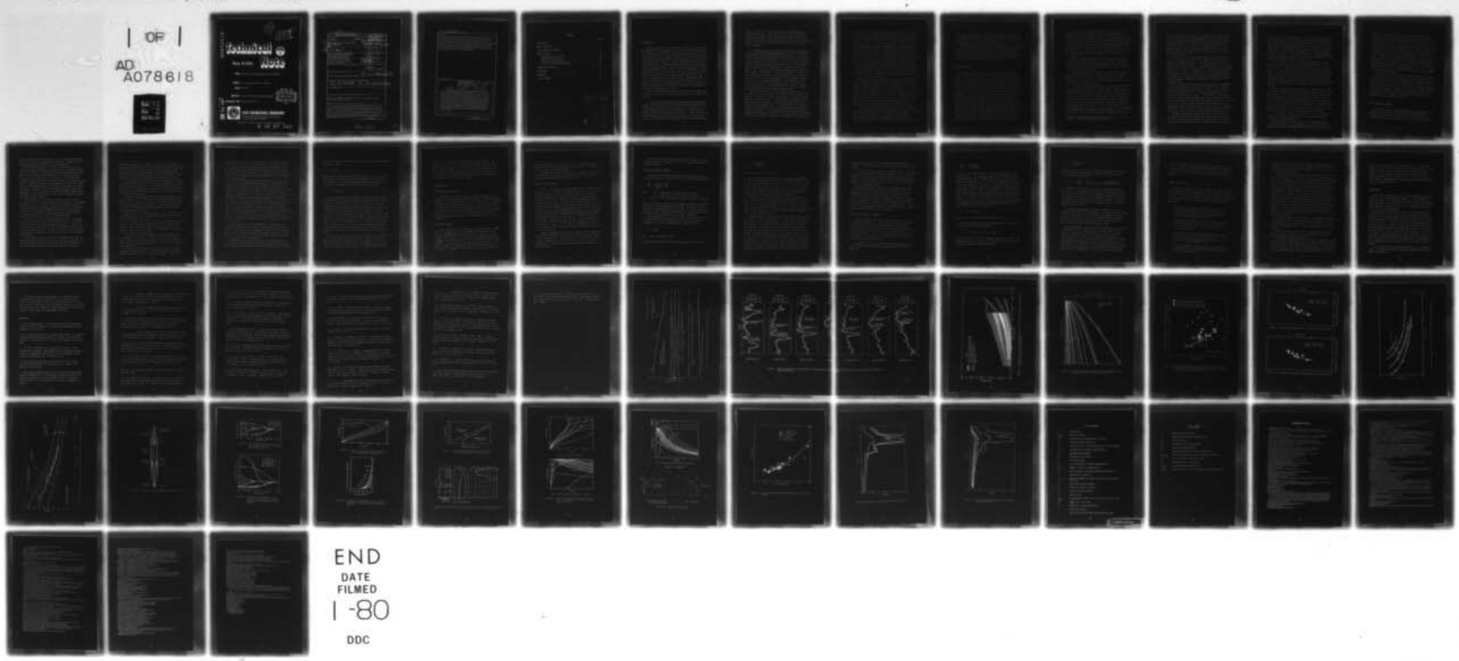

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The vulnerability of waterfront facilities to earthquake-induced soil liquefaction has been demonstrated during several recent earthquakes. A liquefaction threat analysis conducted by the Navy has suggested, however, that reliable procedures for precisely evaluating the extent of this hazard to waterfront structures are not currently available. As a part of a program to remedy this problem, the earthquake-induced liquefaction potential at a coastal Naval installation was evaluated by means of cyclic triaxial testing of ~~(continued)~~ Cont

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INTRODUCTION

Recent earthquakes, particularly those in Alaska, Japan, and Chile, have emphasized the high damage threat the soil liquefaction phenomenon poses to waterfront structures. These experiences have shown that both the nature of waterfront facilities, such as earth-retaining structures, and the depositional environment of the coastal marine soil contribute to major liquefaction damage. A recent study conducted by the Office of Naval Research (Ref 1) recognized a major liquefaction hazard existing at West Coast Naval Stations. A more recent investigation at the Naval Air Station (NAS) North Island, Calif. (Ref 2), concluded that liquefaction under design earthquake levels could result in destruction of such critical structures as aircraft carrier berths, aviation fuel tank farms, and underground utility service lines. Unfortunately, almost all previous studies of the liquefaction problem have been concerned with either conventional building foundations or with analyses of dams, and extensive information is not available on the effect of liquefaction on these types of specialized structures.

Basically two approaches are used for evaluating the liquefaction potential of a deposit of saturated sand subjected to earthquake shaking (Ref 3).

1. Field Data of Performance of Sand Deposits in Previous Earthquakes. Post earthquake surveys of areas where liquefaction has or has not occurred have been used to prepare charts, based primarily on the Standard Penetration Resistance of the deposit, for differentiating between liquefiable and nonliquefiable conditions. Empirical comparisons and evaluations of this type do not take account of such significant factors as the duration of shaking or the extent of drainage and depend

upon the reliability of field observations and such field tests as penetration resistance (often after the fact). Thus, many engineers feel that such correlations provide only preliminary evaluations of liquefaction potential. These, they feel, will often need to be supplemented by detailed studies based on ground stress analyses and soil testing programs.

2. Evaluation of Stress Conditions in the Field and Determination of the Stress Conditions Causing Liquefaction of Soils. Analytical procedures for evaluating the liquefaction potential of soil deposits involve two independent determinations: (a) an evaluation of the cyclic stresses induced at different levels in the deposit by the earthquake shaking and (b) investigation to determine the cyclic stresses that, for given confining pressures representative of specific depths in the deposit, will cause the soil to liquefy or undergo various degrees of cyclic strain. The evaluation of liquefaction potential is then based on a comparison of the cyclic stresses induced in the field with the stresses required to cause liquefaction - or an unacceptable limit of cyclic strain - in representative samples in the laboratory. This approach raises questions regarding the validity of the laboratory results with respect to field conditions and is essentially limited to level ground situations away from load discontinuities.

The Navy has many facilities on reclaimed or marginal land in waterfront locations. The types of structures the Navy builds on such lands (e.g., quay walls, drydocks, storage tanks, etc.) are not widely studied in the civilian sector. Current analytical techniques are not adequate regarding analysis of conditions other than those involving level ground away from structures. The uncertainty associated with soil strength and earthquake occurrence is not routinely used to estimate liquefaction potential. It is apparent that the precision with which liquefaction investigations can be carried out, particularly with regard to ocean front facilities, leaves much to be desired. Better evaluation

techniques must be developed for determining liquefaction hazard in the marine depositional environment existing at many Naval establishments.

One of the major shortcomings in available liquefaction evaluation technology is related to the lack of validated or, in some cases, even credible procedures. It is apparent that realistic large-scale tests for investigating the liquefaction phenomenon are desirable. However, because of the greater number of complex variables which control liquefaction potential, the results to be derived from an individual large-scale test at this time might not warrant the very large expenditures that the Navy would incur. This study, therefore, is currently examining the overall field of liquefaction prediction and evaluation and concentrating upon a broad program on the feasibility of adapting available and anticipated liquefaction analysis developments to Navy requirements.

This study concentrates on two main aspects of liquefaction research: (1) better site evaluation technology and (2) realistic liquefaction analytical techniques. Earthquake prediction and ground motion studies are outside the scope of this project.

NAS North Island, which had recently been designated as having a potential liquefaction hazard (Ref 2), was selected for further study. Several problem soils at this facility were sampled using fixed piston sampling techniques; the most critical soil was subjected to cyclic triaxial testing. The results of this testing procedure are discussed in the next section of this document. The North Island site will be used for further evaluation of soil reconnaissance techniques applied to identifying liquefaction potential. In order to carry out this program more effectively, CEL is now able to conduct cyclic triaxial tests. This will permit in-depth evaluations of the liquefaction potential of characteristic waterfront soil deposits using the current technology.

Analytical procedures for predicting liquefaction response have been improved by use of effective-stress computer code analysis. Computer codes are discussed further in a later section of this document. A state-of-the-art guide (Ref 4) dealing with structures located in earthquake-prone areas has been published for use by the Naval Shore

Establishment. This guide gives general background data on the liquefaction problem and provides a summary of current technology. Reference 4 was excerpted largely from a more comprehensive series of reports completed for the Federal Highway Administration (Ref 5-7).

The objective of the research program discussed herein is to develop procedures for reliably and quantitatively evaluating the threat levels posed by earthquake-generated soil liquefaction at waterfront facilities. These procedures and any associated instrumentation development will be suitable for incorporation into a Navy manual for designing new facilities and taking threat-mitigating measures for existing ones.

NORTH ISLAND SOILS

During the investigation of the earthquake-induced liquefaction threat at NAS North Island (Ref 2), several uncommon soils were encountered. Portions of the Air Station originally below sea level had been reclaimed, using hydraulic fills - primarily silty sands - dredged from the bottom of San Diego Bay. These reclaimed areas had been subjected to several series of filling, most between 1919 and 1952. As a result of investigations made on North Island in connection with foundations of existing structures, a recently filled region was selected for further study (Ref 2). Available soil data were supplemented by additional soil investigations, including dynamic split spoon penetration tests and static cone penetrometer soundings. The split spoon penetrations were conducted as specified in ASTM D 1586 except that a 2-1/2-inch (California-type) split spoon was used. The cone penetration soundings conformed to ASTM D 3441-75T. Based upon this information the generalized soil profile shown in Figure 1 was developed. This soil profile encompasses an old filled bay channel, formerly known as Spanish Bight, which was hydraulically filled to its present elevation in 1945. The old bay bottom immediately prior to the first filling is represented by layer 5.

The results of split spoon penetration tests and friction cone soundings, denoted as penetration holes P1 through P5 along the soil profile of Figure 1, are shown in Figure 2. It should be noted that the split spoon penetration values near the boundary between layers 4 and 5 fall as low as one or two blows per foot. Figure 3 (Ref 3) shows plots of standard penetration test results (number of standard hammer blows required to drive a standard sampler 12 inches) versus in situ soil relative density. For a granular soil at confining pressures of 5 to 10 psi, Figure 3 suggests a relative density,* D_r, for soil stratum 5 of less than 50%.

In Figure 4, Schmertmann (Ref 9) presents relationships between relative density and static cone penetration resistance. With the relatively low friction cone penetration readings of 5 to 10 kg/sq cm shown in Figure 2, values of relative density on the order of only 20% or 30% are indicated for stratum 5.

As suggested in Reference 2, natural soils with relative densities of less than 40% are expected to be quite rare in nature. However, based upon the available information it was necessary to assume, for analysis purposes, that the soil in the region of layer 5 could have a relative density no greater than 35%.

The resistance of granular materials to cyclically induced liquefaction is a function of the in situ relative density. Relative densities less than about 45% are generally considered quite critical and can permit disastrous liquefaction failures at fairly low ground acceleration levels. For example, Figure 5 shows a record of previous liquefaction occurrences as a function of horizontal shear stress over confining stress ratio and relative density (Ref 10). This figure suggests that for relative densities on the order of 40%, stress ratios of about 0.1 would probably result in liquefaction. A stress ratio of 0.1 corresponds to horizontal ground accelerations of about 0.08 (Ref 11) times the acceleration of gravity (or modified Mercalli intensities of VI or VII).

*Relative density is a measure of the present density of a soil with respect to its maximum and minimum densities.

Since most regions in California can be expected to experience ground motions of this severity relatively often (in the geologic time scale), it was considered critical that the liquefaction potential of this soil be explored further. This situation is further amplified when one realizes that this type of coastal soil commonly referred to as "bay deposits" seems to be fairly widely distributed along the coast of Southern California and appears at other Naval facilities such as Long Beach Naval Shipyard.

Therefore, a program of undisturbed sampling and laboratory cyclic triaxial testing was carried out. The sampling program consisted of obtaining soil samples along the soil profile shown in Figure 1 using an Osterberg piston sampler (Ref 12). This sampler utilizes a thin-walled sampling tube 3 inches in outside diameter, 2.88 inches in inside diameter, and 36 inches in length. A complete description of this sampling and testing program is presented in Reference 13. The undisturbed samples were handled and transported to the laboratory using approved techniques. From these samples, characteristic specimens were selected for cyclic triaxial testing.

The soil specimens were subjected to the same level of confinement as the estimated initial in situ vertical effective stress, and then saturated to "B-values" greater than 0.97 prior to testing. Figures 6 and 7 present the results of cyclic triaxial testing on eight soil specimens extracted from soil layer 5. Two criteria for soil failure were used. Figure 6 presents maximum shear stress to confining stress ratio versus the number of load cycles that result in an axial or vertical strain of 5%. Figure 7 presents the same type of information but is based upon the time at which measured pore water pressure in the specimen equals the triaxial confining or chamber pressure (rather than the 5% strain level). Both criteria lead to the same result in this case.

In Figure 8 are shown the North Island cyclic triaxial data for soil layer 5 plotted on top of shake table data presented in Reference 10. The triaxial data have been plotted in terms of the same stress ratio used for the shake table tests. The shake table test results show

the stress ratio versus the number of cycles leading to initial liquefaction (pore pressure in the specimen equals the external confining pressure) for sands at four different relative densities. It is noted that the North Island data plot somewhere above the D_r = 68% curve for the sand in the low cycle range and above the 54% relative density line for the higher cycle range. Thus, although the North Island soil appeared to have a D_r less than 35%, based upon quasi-static and dynamic penetration tests, it exhibits a resistance to liquefaction under cyclic triaxial conditions comparable to that of a denser soil.

This increased liquefaction resistance is apparently due to the distinct structure of the sensitive silty sandy materials in layer 5. This conclusion is further supported by the trend in the test data in Figure 8. It is noted that under higher stress levels (lower number of cycles to failure) the North Island soil performs like one with a higher relative density than like one under lower stress levels (higher number of cycles to failure). Possibly, the longer time to failure under the lower stress levels permits more disturbance of the initial soil structure and, hence, causes the soil to perform as if it had a lower relative density.

Figure 9 portrays some of the results of a compendium of soil data compiled from a number of commercial testing laboratories (Ref 7). The data in Figure 9, for both uniformly graded (SP) and silty sands (SM), have been normalized in terms of D_r. (It has been observed that by dividing the stress ratio causing liquefaction at a particular stress ratio by relative density, reasonable agreement could be obtained between the liquefaction resistance of similar soils at different densities, as presented in Reference 14. Also shown in Figure 9 is the best fit curve (Ref 15) for data from a number of laboratory research programs.

It is noted that in spite of any differences in precision between research and commercial testing activities, the mean soil strength curves are in very good agreement.

Superimposed upon the data in Figure 9 are the North Island soil test data from Figure 7, with an assumed D_r = 60%. The agreement with the mean curves indicates that the North Island soil had a resistance to

liquefaction roughly equivalent to that of a typical soil having a relative density of 60%. The lower D_r values estimated for this soil (35%) based upon the penetration readings could be explained in terms of the sensitivity of the soil structure. It would appear that the soil structure is largely lost during the penetration tests in advance of the sounding device. Thus, the penetration tests, both the split spoon and the friction cone, do not recognize the inherent resistance to cyclic liquefaction provided by the soil structure under cyclic triaxial conditions. Therefore, the most critical soil stratum in the profile in Figure 1 might better be characterized as that having the strength of a sand with a D_r of 60%, rather than the 35% originally assumed. This, in effect, would almost double its liquefaction resistance. This does not necessarily mean that the soil profile in question does not still represent a liquefaction hazard, but rather that the threat may be somewhat less than originally concluded.

It must be kept in mind, however, that should sufficient disturbance occur to destroy the in situ soil structure, the soil stratum in question could perform as the extremely liquefiable material suggested by the penetration results. A significant question that comes up is whether or not the liquefaction resistance as determined by the laboratory specimens is most valid or whether, in fact, the poorer behavior suggested during the penetration tests more realistically predicts the response of this area under an actual earthquake. Unfortunately, the answer to this question cannot be provided at this time.

SITE EVALUATION TECHNIQUES

One of the two major areas being addressed in this program is that of improved reconnaissance or field liquefaction threat evaluation techniques. No matter how sophisticated and reliable analytical procedures may become, they can provide accurate response predictions only

when valid input data can be obtained. In the case of minor structures, where involved analytical programs are not possible, expedient field evaluation techniques become the only available approach.

The liquefaction potential of a soil is directly related to its volumetric change tendencies; more specifically to its volumetric-strain/shear-strain coupling. Traditionally, the volume change properties have been the least studied from the standpoint of in situ measurements. Settlement analysis, predictions of swelling, or other volume change calculations have generally been based upon laboratory testing of acquired samples. Although this may have proven adequate in the past, the recent awareness of earthquake-induced liquefaction has introduced more refined requirements. Problems associated with sample disturbance, laboratory simulation of in situ stress state, temperature, chemical and biological environments, and soil heterogeneity, e.g., severely impair any analysis based upon laboratory testing.

Thus, although in situ testing is attractive in theory, direct determination of in situ volume change properties applicable to earthquake-type loading is not yet available. Recourse must be made to empirical correlations between volume change properties and some form of expedient field test. Various forms of penetration tests have been utilized in this regard, particularly the standard penetration test (ASTM D 1586) and the friction cone penetration test (ASTM D 3441-75T).

Unfortunately, the penetration resistance of a soil is a function not only of its volume change characteristics but also of its strength, shear stiffness, and other deformational characteristics. Penetration resistance can be influenced by factors other than those directly influencing liquefaction potential. Therefore, it may be necessary to measure several types of response before accurate penetration test correlations become possible.

One promising device in this regard is a piezometer probe (Ref 16) which measures pore pressure in the tip of a soil penetrating cone. If, during cone penetration, positive (increased) pore water pressures are generated, then the effective stresses and, hence, the strength and

resistance to penetration are reduced. Alternatively, if negative (reduced) pore water pressures occur, then the soil structure is dilating, effective stresses are increased and so is penetration resistance. The measured incremental changes in pore pressure during penetration of saturated soils are directly related to volume change tendency. They are also a function of soil permeability and rate of penetration.

The rate at which pore water pressures reach equilibrium following cessation of penetration are direct functions of permeability. Thus, a device exists with the potential for correlation with both volume change characteristics and permeability. These two factors (along with nature of loading) are the major determinants which control the occurrence and severity of soil liquefaction.

A schematic of the piezometric probe (Ref 16) developed by Wissa is shown in Figure 10. The rapid response is made possible by means of a high air entry, stainless steel, porous cone tip hydraulically connected to a flush diaphragm pressure transducer. A four-conductor shielded cable encased in polyethylene tubing communicates with the surface through standard "A" drill rods.

Figures 11 and 12 show typical piezometric output readings. Figure 11 shows differences in pore pressure generation and dissipation rates among four different types of soil.

It is noted that loose soils 1 and 2 tend to compact during yielding caused by cone penetration and, hence, generate increased pore water pressures. These induced pore water pressures then dissipate at two different rates: the sand in a matter of a few minutes and the clayey silt over a period of almost 2 hours.

Denser soils 3 and 4 dilate during penetrometer-induced yielding, causing reduction in pore water pressures below that of the datum. Again, stabilizing of pore water pressure is a function of permeability, with silt requiring about 30 times longer for pressure equalization than dense sand requires.

A comparison of the behavior of similar soil types at different densities is shown in Figure 12. As hypothesized, soils in the dense state dilate, whereas those in the loose state compact. The time of

pore pressure equilization is shown to be both a function of soil type (grain size) and state of initial compactness; (i.e., loose or dense).

Obviously, substantial differences exist between stresses induced by seismic events such as earthquakes and stresses that occur during cone penetrations. Nevertheless, since responses in both cases are a function of soil volume change tendency and permeability, useful correlations may be possible.

An entirely different approach to liquefaction prediction is one based upon soil resistivity measurements (Ref 17). The liquefaction potential of a saturated soil is largely influenced by the state of compactness (or relative soil density). However, many additional factors can have an influence here. One term used in conjunction with the strength of cohesionless soils is "fabric." This refers to the structure, or the way in which the individual grains interlock or fit together. The fabric of a soil is largely controlled by the way in which the soil was originally formed and its subsequent stress or deformational history. This factor has been investigated recently by considering different types of sample preparation and conditioning (Ref 18-21). Substantial results now exist to support the fact that soil fabric exerts a measurable effect on soil liquefaction potential, at least in the laboratory. Thus, if it were possible to measure soil density and fabric in situ, it is conceivable that greatly improved liquefaction prediction capability would be available.

A device being applied by Arulanandan (Ref 17) appears to have considerable potential in this regard. This approach, by measuring conductivity in two directions, attempts to characterize the soil structure to account for shape and spatial arrangement of particles. Thus, void ratio and a measure of structural anisotropy can be deduced which can be related to cyclic loading behavior.

The approach utilizes a parameter called the formation factor, F, defined as the ratio of conductivity of the electrolyte (pore water) to the conductivity of the sand saturated by the electrolyte. Considerable

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data exists (Ref 22 and 23) supporting the following relationship between porosity, n, and F:

$$F = \frac{3 - n}{2\,n}$$

Other work (Ref 24) has extended this relationship to include a particle shape factor, x, which is a measure of the sphericity of soil grains. For example, x = 2 for perfect spheres and decreases as the axial ratio of the spheroid increases. This relationship is of the form

$$F = \frac{(x + 1) - n}{xn}$$

The problem with such relationships is that a correlation for the particular sand had to be established first (generally in the laboratory), and this failed to preserve the initial in situ fabric. More recent developments at the University of California at Davis (Ref 17) are designed to bypass this problem. This work hypothesizes that the resistivity path involved in measurement of F is a function of particle orientation. Thus, by measuring an anisotropic index, A (defined as the square root of the ratio of vertically measured to horizontally measured formation factors) in situ, a measure of the natural soil fabric can be measured, prior to any sampling disturbance. There are still problems to be addressed such as the effects of any interparticle cementations and assessing of the in situ lateral pressures. Nevertheless, this approach does show promise of providing valuable in situ definitive soil data.

Various other devices and procedures are available for determining in situ volume change tendencies and, hence, liquefaction potential. These include borehole permeability, soil pressuremeter, cyclic screw plate, and other tests; however, these test methods are not considered suitable for further discussion herein. The borehole permeability test

concerns itself primarily with long-term consolidation effects. The other two tests, even if combined with pore pressure readings, would be difficult to interpret from a liquefaction evaluation viewpoint. Other more esoteric techniques are available, such as the water cannon (Ref 25) developed at the Technical University at Zurich or the cylindrical in situ (CIST) test (Ref 26), but such tests are not considered compatible with the aims of this research program.

COMPUTER MODELS

Advanced Analytical Models

Significant research is in progress to model the generation and dissipation of pore pressure with seismic excitation. Present implemented technology is essentially limited to total stress analysis (i.e., ignoring the two-phase, soil-pore fluid interaction): however, current research is studying effective stress techniques. The transition from both total stress analysis and effective stress analysis to a tool that goetechnical engineers can use to solve practical design problems is not imminent. The concept, however, represents a critical advance which could be very significant.

Total Stress Analysis

Several techniques are available for total stress analysis. SHAKE (Ref 27) and APOLLO (Ref 28) (also GADFLEA, Ref 29) have been used together. SHAKE generates total stress histories that are converted to equivalent stress cycles. These data and the soil strength are used to determine the undrained number of cycles to liquefaction, which is input to APOLLO. A one-dimensional code, APOLLO, and a two-dimensional code, GADFLEA, compute the uncoupled pore pressure buildup and dissipation. No provision is included to account for variation of soil modulus with

increasing pore pressure and reduced effective confinement. SHAKE uses equivalent linear soil properties, which are adjusted for strain amplitude. An example is illustrated below.

Narasimhan (Ref 30) uses a seepage program - TRUST - to analyze the dissipation phase of pore pressure production. The initial pressures are estimated by undrained total stress analysis.

Effective Stress Analysis

Finn et al. (Ref 31) have developed DESRA which is a one-dimensional, effective-stress, soil model that computes pore pressure. This is a nonlinear procedure in which the equations of motion are integrated using the Newmark method. The initial loading is given by a hyperbolic stress-strain relation. Reloading, described by the Masing criterion, produces hysteresis loops. The upper curve of the hysteresis loop can be obtained from the loading curve by translating the origin and increasing the scale vertically and horizontally by a factor of two. When used in the effective stress mode, DESRA produces increases in pore pressure with each cycle, using the parameters which describe the volume change and rebound characteristics of the soil. The reduction in effective stress results in reduction of shear modulus. It includes the effects of both shear strain and progressive increase in pore-water pressure on soil properties. However, it is limited to one-dimensional cases as well as equivalent cyclic input (not actual time histories), and it does not treat dissipation.

Ghaboussi and Dikmen (Ref 32) have a program of similar capabilities. This program, although one-dimensional, uses actual time history input as the loading function. The program utilizes a material model similar to that of Ishihara (Ref 33 and 34), modified by an elliptic stress loading path. The program treats only effective stress generation and not dissipation.

Baladi and Rohani (Ref 35) have modified the "cap model" to pore pressure generation. This model is still in the research stage and has not been implemented.

Total Stress Analysis Example

Seed et al. (Ref 28) have investigated the distribution of hydrostatic pore pressure u in the soil by use of the one-dimensional equation

$$\frac{\partial u}{\partial t} = C_v \left(\frac{\partial^2 u}{\partial z^2} \right) + \frac{\partial u_g}{\partial t}$$

where C_v = coefficient of consolidation of the soil

z = depth within soil (the vertical coordinate)

$\partial u_g/\partial t$ = rate of pore pressure generation caused by earthquake

This is the diffusion equation used in Terzaghi's classical consolidation theory, with a pressure-generating term added. The solution of this equation is accomplished by the finite-difference technique using incremental time steps. The pore pressure generation is estimated by Figure 13 as a function of the number of cycles to cause liquefaction.

The coefficient of consolidation C_v, which is defined in terms of the coefficient of volume compressibility m_v and the coefficient of permeability k, may be estimated by means of Figures 14 and 15.

$$C_v = \frac{k}{m_v \gamma_w}$$

where γ_w = the weight of water

The rise in the water table ΔH, during time increment Δt, is given by:

$$\Delta H = \frac{-k\left(\frac{\partial u}{\partial z}\right)\Delta t}{n_e}$$

where n_e = the effective porosity

This procedure has been automated in the computer program APOLLO and may be used with the analysis in the computer program SHAKE. SHAKE is used to produce the equivalent uniform cyclic shear stress, τ_{eq}, and the equivalent number of uniform stress cycles, n_{eq}, for various depths of soil. From strength data the number of cycles to cause liquefaction at each depth is determined. Using this information, program APOLLO solves the pore pressure generation-dissipation equation.

The pore pressure generation function is based on undrained test data. This application is deemed sufficiently accurate when small time steps are used to properly account for drainage. The elastic response analysis used to determine the number of cycles to liquefaction can be made in order to consider the isolation effects of subsurface liquefaction on near surface shaking and the reduction in pore pressure generation when iteration techniques are used.

A typical example from Seed et al. (Ref 28) from the Niigata earthquake of 1964 is shown in Figures 16 and 17. The computed variations of pore water pressure with time are given. Figure 17 shows the buildup of pore pressures. It may be seen that the sand layer at a depth of 15 feet liquefies after about 21 seconds of shaking; liquefaction extends to depths of 20, 30, and 40 feet after about 23, 32, and 40 seconds of shaking. Although the layers above a 15-foot depth continue to increase in pore pressure as the shaking progresses, the rate of increase is very low after the 15-foot level liquefies. Reference 2 notes that when the pore pressure ratio in the top foot of soil reaches 60%, the ground will become soft, and a man will sink. This occurred after about 8.5 minutes in Niigata, determined in the analysis. The pore pressure ratio at the ground surface begins to decrease after about 20 minutes but will not

support a man until about 40 to 50 minutes after the earthquake. The results of the computer analysis are in general agreement with observed reports.

If the water table were located at a depth of 15 feet, no significant pore pressure increases would occur in the upper 10 feet of soil, even though the soil is liquefied between 15 and 40 feet. Thus, in this situation the bearing capacity of small shallow footings near the surface might well be essentially unaffected by the dissipation of pore water pressures in the liquefied zone.

Program APOLLO has been expanded into a two-dimensional computer program called GADFLEA. The approach is very similar to the one-dimensional analysis, requiring as input information the number of cycles causing liquefaction by soil element. The number of cycles causing liquefaction is a function of the applied shear stress loading and soil confinement. These may be determined from a conventional two-dimensional elastic or inelastic finite element analysis. Using the input data program, GADFLEA computes the two-dimensional pore pressure generation and dissipation from the earthquake.

Effective Stress Analysis Example

Ghaboussi and Dikmen (Ref 32), using their effective stress model, also studied the Niigata site. Ghaboussi uses two distinct material behavior models to cover the range from the initial in situ stress condition to liquefaction. The first model represents the soil behavior from the in situ stress condition up to the initial liquefaction. A second material model is used to treat post-liquefaction. The initial liquefaction is to be considered a condition of near failure (not 100% pore pressure confining stress ratio).

Preliquefaction. The shear stress-strain relationship under monotonic loading is represented by the following equation and shown in Figure 18a.

$$\left(\frac{q}{p'}\right) = \frac{\gamma G_o S_{max}}{\gamma G_o + S_{max}}$$

in which q is the shear stress, p' is the effective pressure, and γ is the shear strain. The factors S_{max} and G_o are defined in the figure. The unloading takes place linearly with the slope G_o until previous maximum or minimum value of q/p' is reached, whereupon the above stress-strain relationship becomes valid again. The relationship is active up to the onset of initial liquefaction. Experimental evidence (Ref 33) suggests that in the p'-q plane the shear yield loci (lines of equal shear strain) take the form of straight lines radiating from the origin (Figure 18b). At higher values of effective pressure, the yield loci approach a state parallel to the effective stress (p') axis. Under monotonic shear stress increase, failure occurs on a line corresponding to an asymptote of the shear stress relationship indicated earlier. This critical state line f_1 is given by

$$f_1 = q - p' \tan \phi = 0$$

where $\tan \phi$ = the frictional strength of the soil

The stress path to failure f_2 has been approximated by a quarter of an ellipse given by the following equation.

$$f_2 = (p' - p'_f)^2 + \frac{1}{\lambda 2} q^2 - (p'_o - p'_f)^2 = 0$$

where the subscript f is used to designate conditions of failure. The material parameter λ is the ratio of the major and minor axes of the ellipse, as given by the following relation.

$$\lambda = \frac{p'_f}{p'_o - p'_f} \tan \phi$$

The stress path f_2 is completely defined by the two material parameters λ and ϕ and the in situ effective pressure p'_o and can be written in terms of the following parameters.

$$f_2 = q^2 + \lambda^2 \left[p'^2 - \left(\frac{2\lambda}{\lambda + \tan\phi} \right) p' \, p'_o + \left(\frac{\lambda - \tan\phi}{\lambda + \tan\phi} \right) p'^2_o \right] = 0$$

The effective pressure p' decreases as a result of increases in pore pressure. In unloading, the effective pressure remains constant until the stress path crosses previous maximum or minimum value of a/p', at which point a new value of p'_o is determined, using the f_2 function. The parameter λ is related to relative density (Figure 19). Figure 19 is based on fairly uniform sands with rounded particles. Variation in shape has some effect on λ.

Post Initial Liquefaction. The material behavior of the soil changes abruptly after initial liquefaction. The stress path is defined, based on very limited soil data, but further development is required in this area of investigation. After the earthquake motion has ceased, excess pore pressures exist in the soil layers. These dissipate with time and require analysis using a dissipation program.

Niigata Case Study. The Niigata site discussed earlier was studied by Ghaboussi (Ref 32). Figures 20, 21, and 22 give results of that analysis. The effective stress analysis shows that the increase in effective pressures with depth have a significant influence on the maximum shear stresses, equivalent shear moduli, and hysteretic damping. Although modelling differences do exist, the qualitative behavior is very similar to that of the total stress analysis in predicting the zone that will undergo liquefaction. However, an effective stress analysis allows for strain softening not possible with a total stress analysis.

This may be significant when isolation from shaking of upper layers occurs as a result of weakened (liquefied) deeper layers. Though much additional work is required on this model, it appears to have great potential. It is presently limited to one-dimensional preliquefaction analysis.

SUMMARY OF PAST EFFORT

Technology for defining the soil liquefaction phenomenon is developing through many different approaches. This is in response to the knowledge that soils are a critical constituent of most civil engineering structures. As indicated in Reference 36, poor soil behavior during past earthquakes has led to structure failures. The following are examples.

- Relatively moderate ground shaking in the 1964 Niigata, Japan, earthquake caused extensive ground failure and loss of support for buildings due to soil liquefaction, which resulted in damage approaching $1 billion.

- The catastrophic landslides and other soil failures in the 1964 Alaska earthquake destroyed much of the developed residential and commercial property in several cities as well as many highway bridges.

- During the 1906 San Francisco earthquake, ground cracking and sliding caused rupture of many buried water lines, thus immobilizing fire fighting operations resulting in uncontrolled fire sweeping through the city.

- In 1972 an earthquake in Peru produced an enormous avalanche and landslide which rushed down the mountain and buried entire villages in the valley below.

In addition, two earth dams nearly failed in the 1971 San Fernando earthquake. This failure would have inundated a residential area of 80,000 people and could have been the greatest single natural disaster in the history of our country.

Although all of these tragedies cannot be attributed solely to the liquefaction process, it is widely accepted that soil liquefaction is the major earthquake-associated threat confronting geotechnical engineering. In spite of the increased efforts expended for solution of this problem, it was the consensus reached at a 1977 workshop attended by 72 internationally recognized experts (Ref 36) that available geotechnical earthquake engineering techniques and design procedures are either questionable or inadequate. This evaluation is particularly pertinent to the waterfront area. Marine earthworks, including such waterfront structures as tied bulkheads, cofferdams, rockfill jetties, and partially relieved quay walls, are commonly situated in relatively precarious regions. Often, the soil fill associated with such structures is poorly compacted. Evaluation techniques are necessary for assessing (1) the integrity of soil foundations for waterfront structures and (2) in particular, the susceptibility of the fill materials to loss of stability during seismic loading.

The research program described herein is concerned with technology transfer, as well as with individual or unique developments. As such, it concentrates primarily upon two areas: (1) expedient and reliable site evaluation techniques and (2) valid analytical response prediction procedures.

To date, an appraisal of the best available analytical computer codes for evaluating liquefaction has been conducted. The Navy is offering support to the most promising ideas.

A report has been published (Ref 4) which offers preliminary guidance to the Naval Shore Establishment until more adequate liquefaction technology is available. This report summarizes the current state-of-the-art regarding placement of structures in liquefiable regions. Field studies were continued at NAS North Island, which had previously (Ref 2) been

identified as representative of a liquefaction hazard. Undisturbed samples of waterfront-unique, potentially liquefiable soils were obtained and tested in the laboratory under simulated earthquake conditions (Ref 13). The information secured indicated - at least for the one highly structured coastal soil tested to date - that liquefaction resistance appears to be somewhat greater than that based on correlations derived from simple field tests.

FUTURE WORK

Work for the upcoming year will be directed along three different avenues of study, with roughly equal emphasis: (1) improvement of analytical models, (2) investigation of innovative site reconnaissance techniques, and (3) more detailed studies of soil parameters with particular attention to the Navy's unique soils, using CEL's newly acquired liquefaction testing capabilities.

With regard to the latter area, only one of the problem soils at NAS North Island (a very sensitive coastal silt deposit) has been subjected to cyclic laboratory testing to date and appears to pose a major liquefaction threat. Several other unusual soils exist at this facility. One soil of interest is a hydraulic fill sand containing an unusually high percentage of mica flakes. How this type of dredged fill material will behave under earthquake shaking must be determined. Some forms of parameter studies of these coastal marine deposits must be carried out. For example, it has been observed that when a substantial portion of any granular soil passes the #200 mesh sieve the penetration resistance is drastically reduced in both static and dynamic tests. The significance of this phenomenon should be determined for actual dynamic or cyclic behavior.

It is intended that the most promising devices such as piezometric cones or directional resistivity measuring instruments be evaluated at the North Island site.

With regard to analytical developments, it is intended that the most promising available soil models will be incorporated into CEL's library of finite element codes, which would then be exercised on particular solutions. Eventually such a code would be combined with a groundwater flow code (such as TRUST) for complete definition of soil-structure interaction under dynamic and hydrodynamic conditions.

REFERENCES

1. Office of Naval Research. General review of the seismic hazard to selected U.S. Navy installations, by ONR National Hazards Review Panel. Washington, D.C., Jan 1974.

2. Civil Engineering Laboratory. Technical Report R-847: An earthquake analysis of the liquefaction potential at the Naval Air Station North Island, by J. M. Ferritto and J. B. Forrest. Port Hueneme, Calif., Sep 1976.

3. H. B. Seed. "Evaluation of soil liquefaction effects on level ground during earthquakes," ASCE Preprint 2752 (Liquefaction Problems in Geotechnical Engineering), American Society of Civil Engineers Annual Convention, Philadelphia, Pa., 27 Sep-1 Oct, 1976.

4. Civil Engineering Laboratory. Technical Note N-1530: Seismic liquefaction potential, by J. M. Ferritto and J. B. Forrest. Port Hueneme, Calif., Sep 1978

5. Federal Highway Administration, Offices of Research and Development. Report No. FHWA-RD-77-127: Determination of seismically induced soil liquefaction potential at proposed bridge sites; Vol I, Theoretical considerations, by J. M. Ferritto and J. B. Forrest. Washington, D.C., Aug 1977.

6. __________. Report No. FHWA-RD-77-128: Determination of seismically induced soil liquefaction potential at proposed bridge sites; Vol II, Planning guide for evaluation of liquefaction, by J. M. Ferritto and J. B. Forrest. Washington, D.C., Aug 1977.

7. __________. Report No. FHWA-RD-77-129: A compilation and analysis of cyclic triaxial test data, by J. M. Ferritto and J. B. Forrest. Washington, D.C., Aug 1977.

8. Army Waterways Experiment Station. Research Report S-76-2: Liquefaction potential of dams and foundations; Report 1, Laboratory standard penetration tests on Reid Bedford model and Ottawa sands, by W. A. Bieganousky and W. F. Marcuson. Vicksburg, Miss., Oct 1976.

9. __________. Technical Report S-78-2: Study of feasibility of using Wissa-type piezometer probe to identify liquefaction potential of saturated fine sands, by J. H. Schmertmann. Vicksburg, Miss., Feb 1978.

10. University of California, Earthquake Engineering Research Center. EERC Report No. 75-14: Determination of soil liquefaction characteristics by large-scale laboratory test, by P. DeAlba, C. K. Chan, and H. B. Seed. Berkeley, Calif., May 1975.

11. H. B. Seed and I. M. Idriss. "A simplified procedure for evaluating soil liquefaction potential," Journal of the Soil Mechanics and Foundation Division, American Society of Civil Engineers, vol 97, no. SM6, Sep 1971.

12. W. C. Teng. Foundation design. Englewood Cliffs, N.J., Prentice-Hall Inc., 1962.

13. Civil Engineering Laboratory. Project No. 78-174: Soil sampling and liquefaction testing programs, Naval Air Station, North Island. Long Beach, Calif., Fugro, Inc., Aug 1978. (Contract N68305-8019-0002)

14. H. B. Seed and K. L. Lee. "Liquefaction of saturated sands during cyclic loading," Journal of the Soil Mechanics and Foundations Division, American Society of Civil Engineers, vol 92, no. SM6, Nov 1966, pp 105-134.

15. N. C. Donovan. CUMLIQ: Evaluation of potential for liquefaction of a soil deposit using random vibration procedures. San Francisco, Calif., Dames and Moore, Jul 1974.

16. A. E. Z. Wissa, R. T. Martin, and J. E. Garlanger. "The Piezometer Probe," in Proceedings of the Conference on In Situ Measurement of Soil Properties, Vol I, North Carolina State University, Rayleigh, N.C., 1-4 Jun 1975. New York, N.Y., American Society of Civil Engineers, 1975.

17. K. Arulanandan and B. Kutter. "A directional structure index related to sand liquefaction," in Proceedings of the ASCE Geotechnical Engineering Division Specialty Conference on Earthquake Engineering and Soil Dynamics, Vol 1, Pasadena, Calif., 19-21 Jun 1978. New York, N.Y., American Society of Civil Engineers, 1978.

18. R. S. Ladd. "Sample preparation and liquefaction of sands," Journal of Soil Mechanics and Foundations Division, American Society of Civil Engineers, vol 100, no. GT10, Oct 1974.

19. J. P. Mulilis, H. B. Seed, C. K. Chan, J. K. Mitchell, and K. Arulanandan. "Effects of sample preparation on sand liquefaction," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, vol 103, no. GT2, Feb 1977.

20. Army Waterways Experiment Station. Contract Report S-76-5: The influence of sand fabric on liquefaction behavior, by J. K. Mitchell, J. M. Chatoian, and G. C. Carpenter. Vicksburg, Miss., Jun 1976.

21. M. Oda. "Initial fabric and their relation to mechanical properties of granular materials," Soils and Foundations (Japan), vol 12, no. 1, 1972.

22. G. E. Archie. "The electrical resistivity log as an aid in determining some reservoir characteristics," in American Institute of Mechanical Engineers Transactions, vol 146, pp 54-61, 1942.

23. W. O. Winsauer, H. M. Shearing, P. H. Masson, and M. Williams. "Resistivity of brine saturated sands in relation to pore geometry," Bulletin of the American Petroleum Geologists, vol 36, no. 21, Feb 1952.

24. H. Fricke. "The electrical conductivity and capacity of disperse systems," Physics, vol 1, 1931.

25. E. Barnert, G. Schnitter, and M. Weber. "A field method of determining soil properties by impact loading," in Proceedings of International Symposium on Wave Propagation and Dynamic Properties of Earth Materials, Albuquerque, N.M., 23-25 Aug 1966.

26. J. L. Bratton and C. J. Higgins. "Measuring dynamic in situ geotechnical properties," in Proceedings of the Conference on Earthquake Engineering and Soil Dynamics, Geotechnical Engineering Division, American Society of Civil Engineers Specialty Conference, Pasadena, Calif., 19-21 Jun 1978.

27. University of California, Earthquake Engineering Research Center. EERC Report No. 72-12: SHAKE: A computer program for earthquake response analysis of horizontally layered sites, by P. B. Schnabel, J. Lysmer, and H. B. Seed. Berkeley, Calif., Dec 1972.

28. __________. EERC Report No. 75-26: The generation and dissipation of pore water pressures during soil liquefaction, by H. B. Seed, P. P. Martin, and J. Lysmer. Berkeley, Calif., Aug 1975.

29. __________. EERC Report No. 76-24: GADFLEA: A computer program for the analysis of pore pressure generation and dissipation during cyclic or earthquake loading, by J. R. Booker, M. S. Rahman, and H. B. Seed. Berkeley, Calif., Oct 1976.

30. Lawrence Berkeley Laboratories. LB7-7037: Numerical model for saturated-unsaturated flow in deformable porous media, Part III, Applications, by T. N. Narasimhan and P. A. Witherspoon. Berkeley, Calif., Mar 1978.

31. W. D. Finn, G. R. Martin, and M. K. Lee. "Comparison of dynamic analysis for saturated sands," in Proceedings of the ASCE Geotechnical Engineering Division Specialty Conference on Earthquake Engineering and Soil Dynamics, Pasadena, Calif., 19-21 Jun 1978. New York, N.Y., American Society of Civil Engineers, 1978.

32. University of Illinois. UILUENG 77-2010: LASS II: Computer program for analysis of seismic response and liquefaction of horizontally layered sands, by J. Ghaboussi and S. U. Dikmen. Urbana, Ill., Jun 1977.

33. K. Ishihara, F. Tatsuoka, and S. Yasuda. "Undrained deformation and liquefaction of sand under cyclic stresses," Soils and Foundations (Japan), vol 15, no. 1, Mar 1975.

34. K. Ishihara et al. "Prediction of liquefaction in sand deposits during earthquakes," Soils and Foundations (Japan), vol 16, no. 1, Mar 1976.

35. Army Waterways Experiment Station. An elastic-plastic work-hardening constitutive model for fluid-saturated granular material, by G. Y. Baladi and B. Rohani. Vicksburg, Miss. (To be published)

36. National Science Foundation. "Research needs and priorities for geotechnical earthquake engineering applications," Proceedings of Workshop held at University of Texas, Austin, Tex., 2-3 Jun 1977. (NSF Grant No. AEN 77-09861).

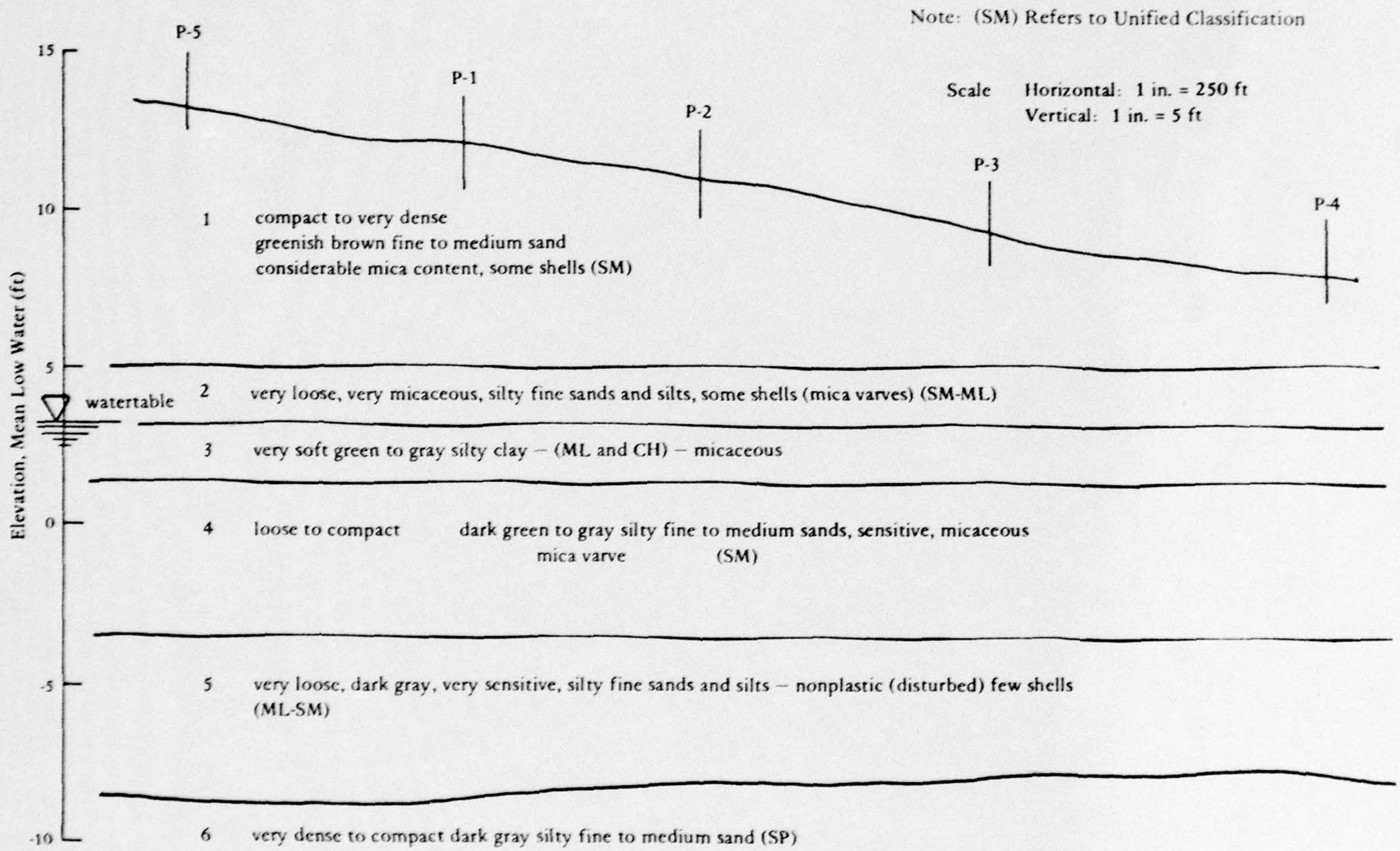


Figure 1. Generalized soil profile across former Spanish Bight.

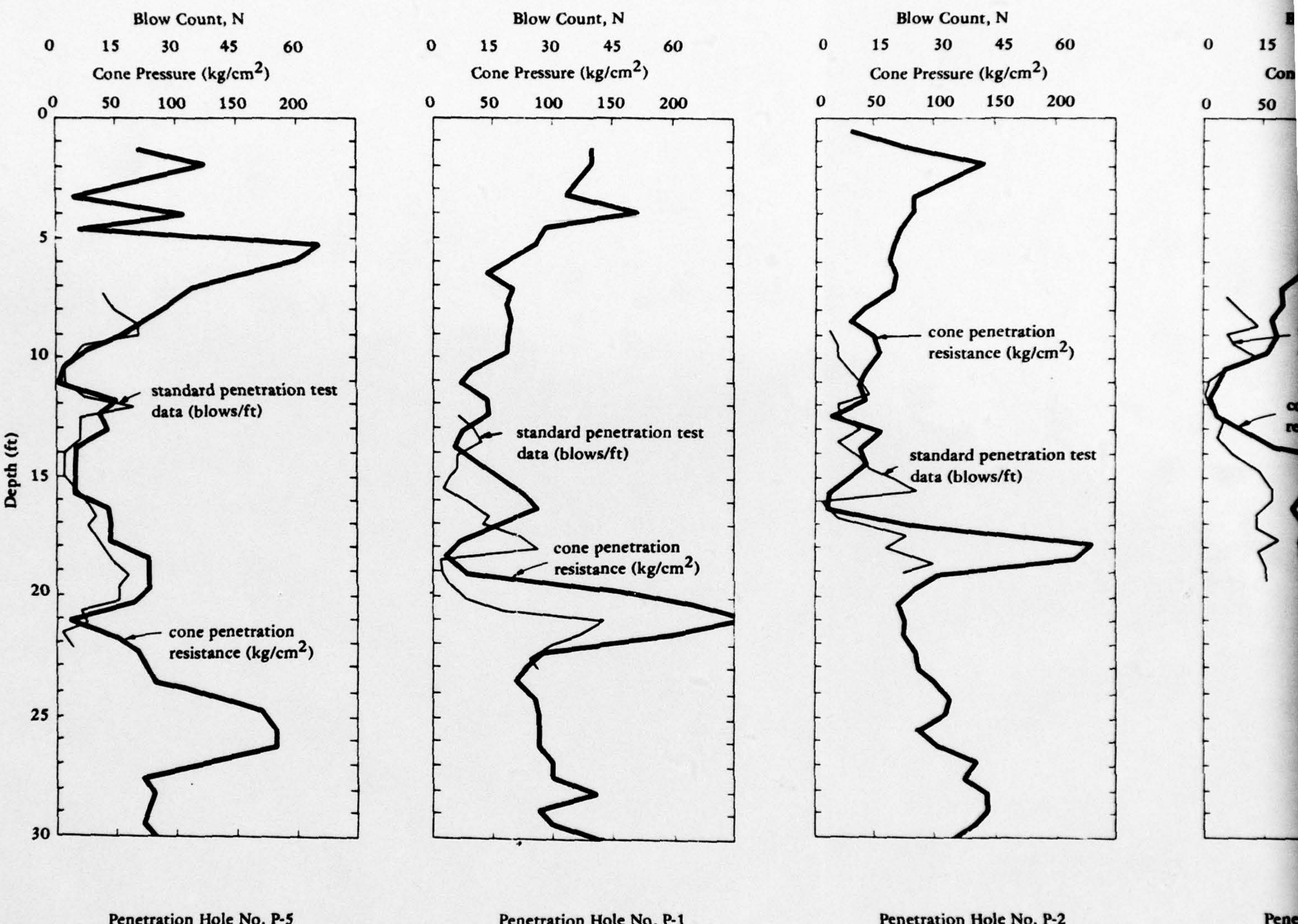
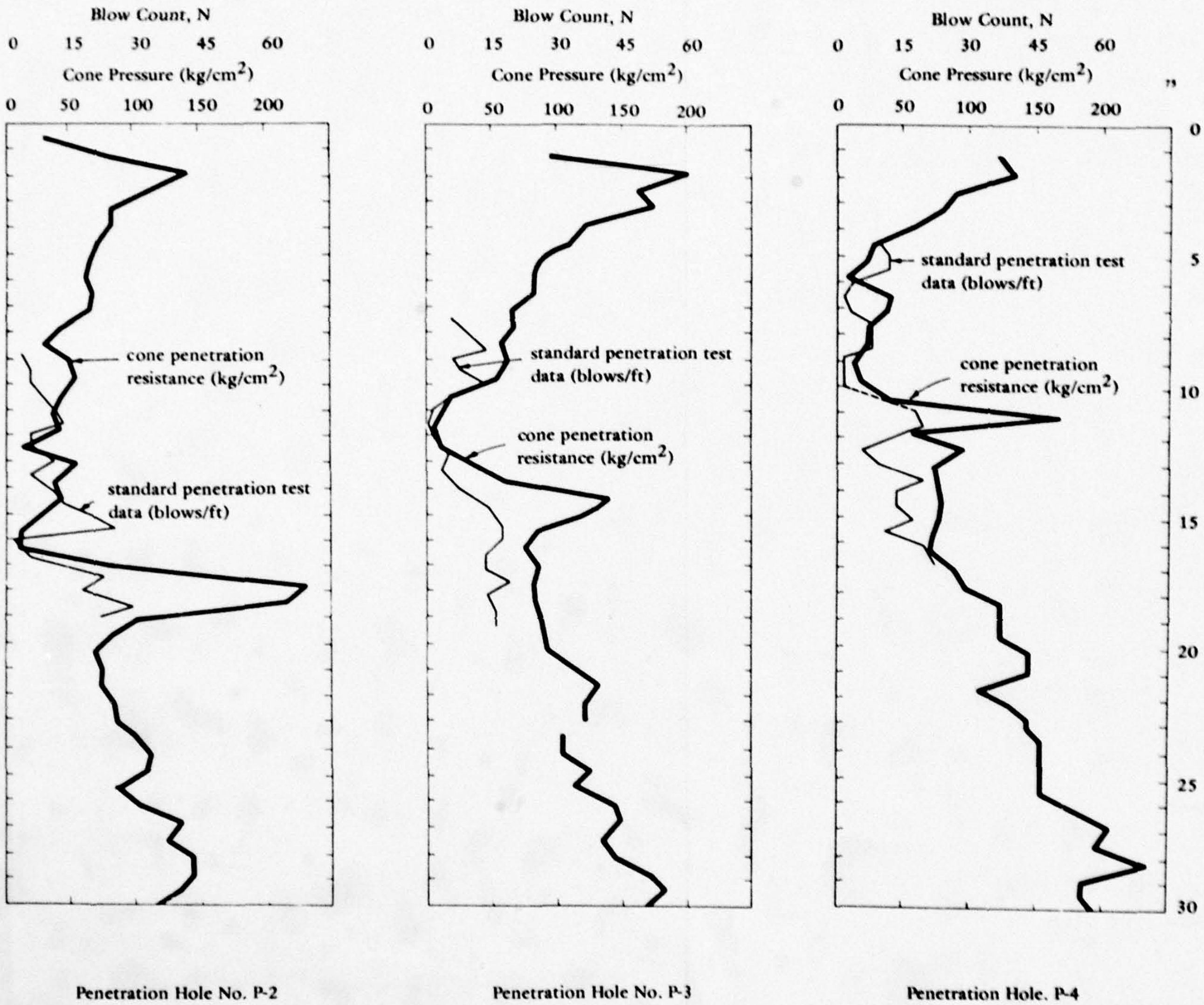


Figure 2. Dynamic split-spoon and quasi-static cone penetration resis
North Island soils.



poon and quasi-static cone penetration resistance of
ils.

2

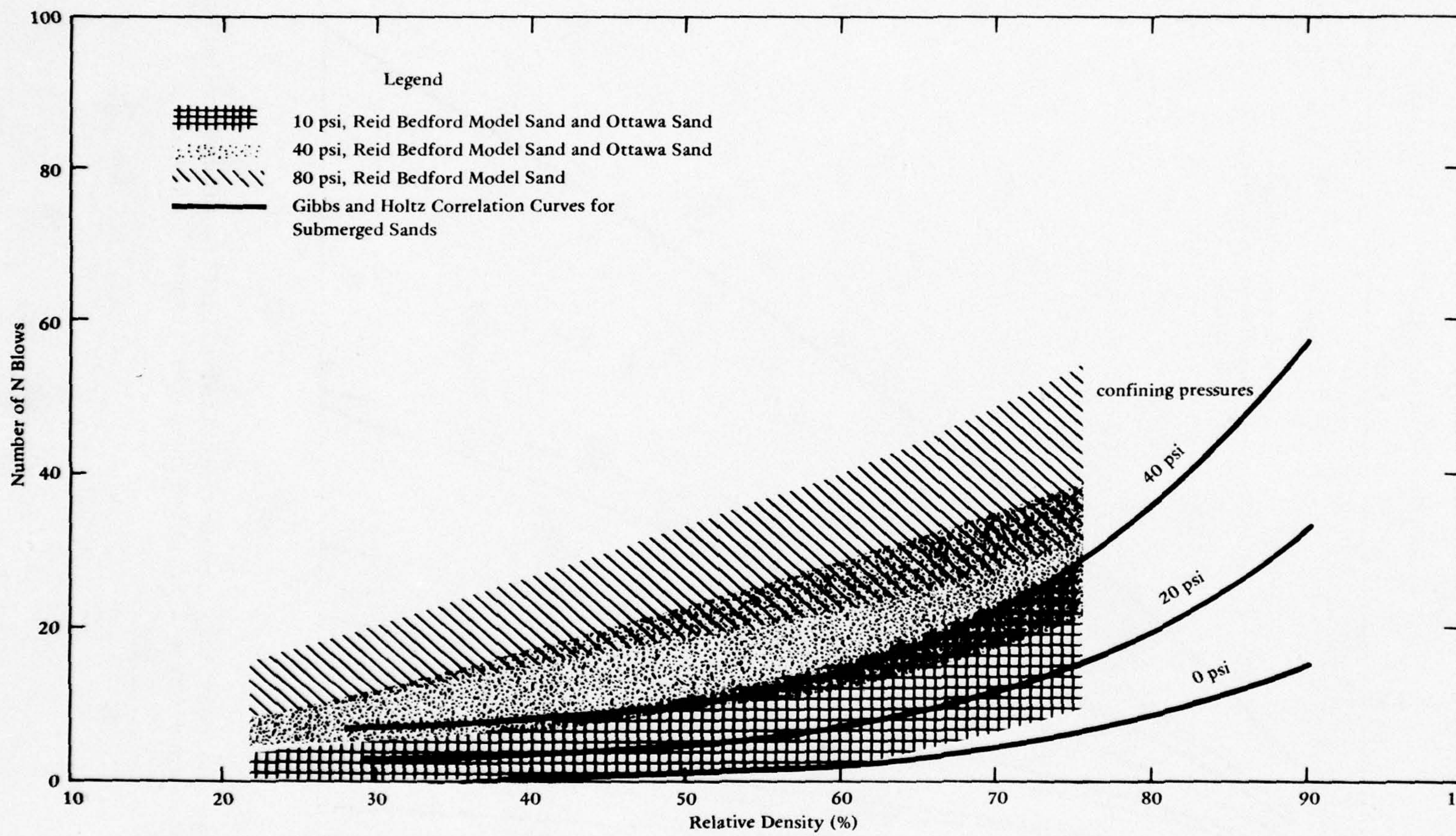


Figure 3. Comparison of Waterways Experiment Station data and Gibbs and Holtz correlation curves for submerged sands (Ref 8).

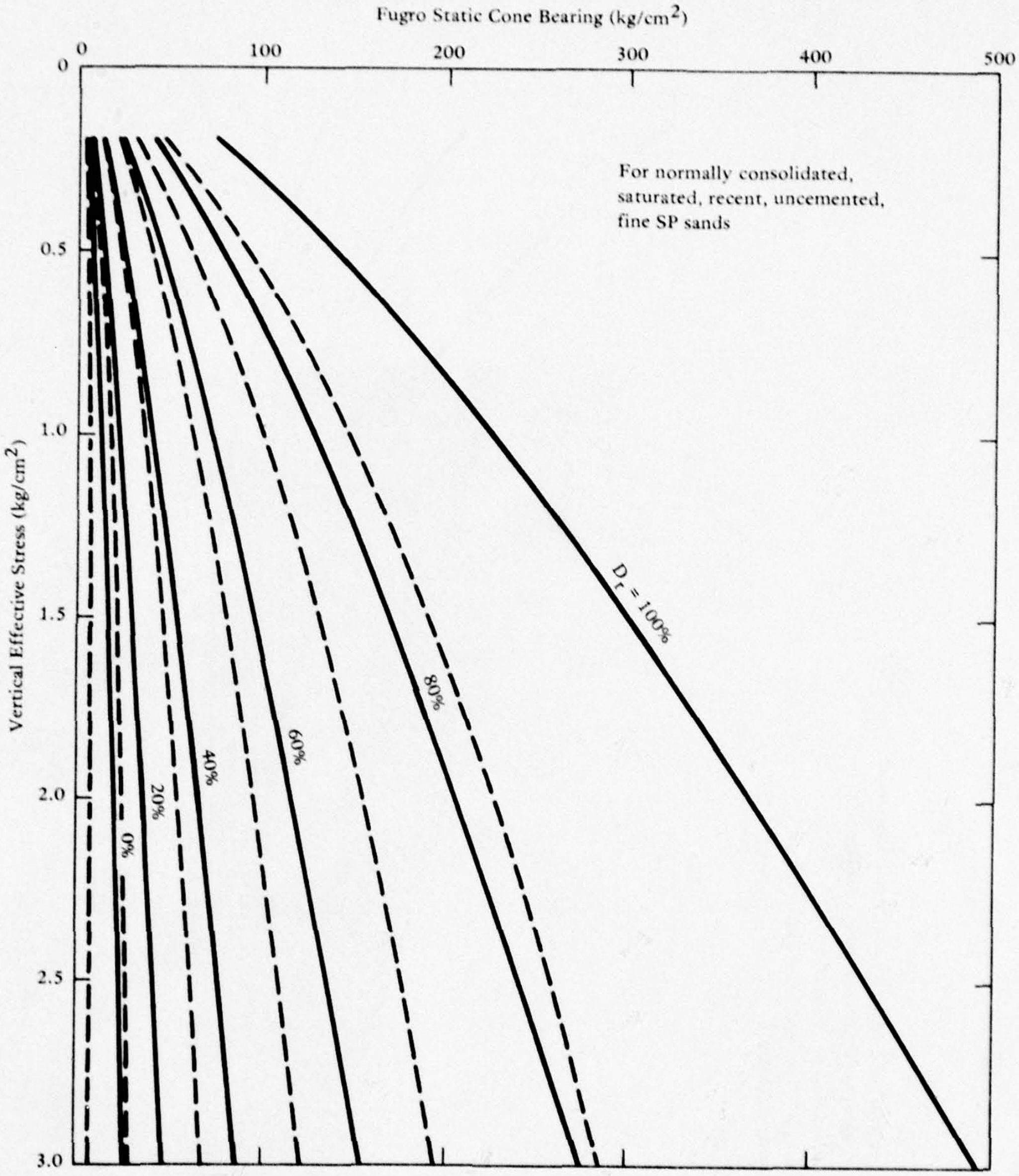


Figure 4. Correlations between vertical effective stress, relative density, and friction cone resistance (Ref 9).

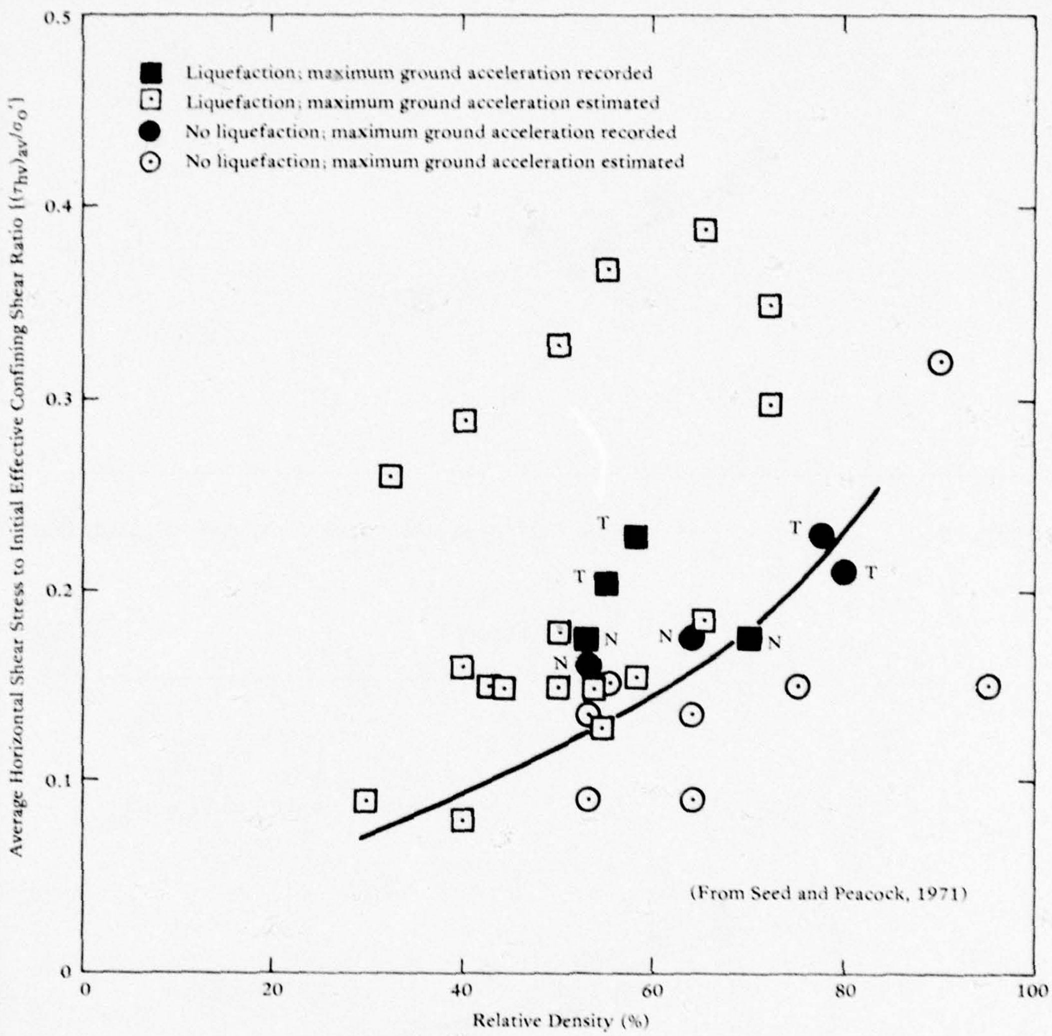


Figure 5. Relationship between shear stress/confining stress ratio $[(\tau_{hv})_{av}/\sigma_o']$ and relative density for known cases of liquefaction and nonliquefaction (Ref 10).

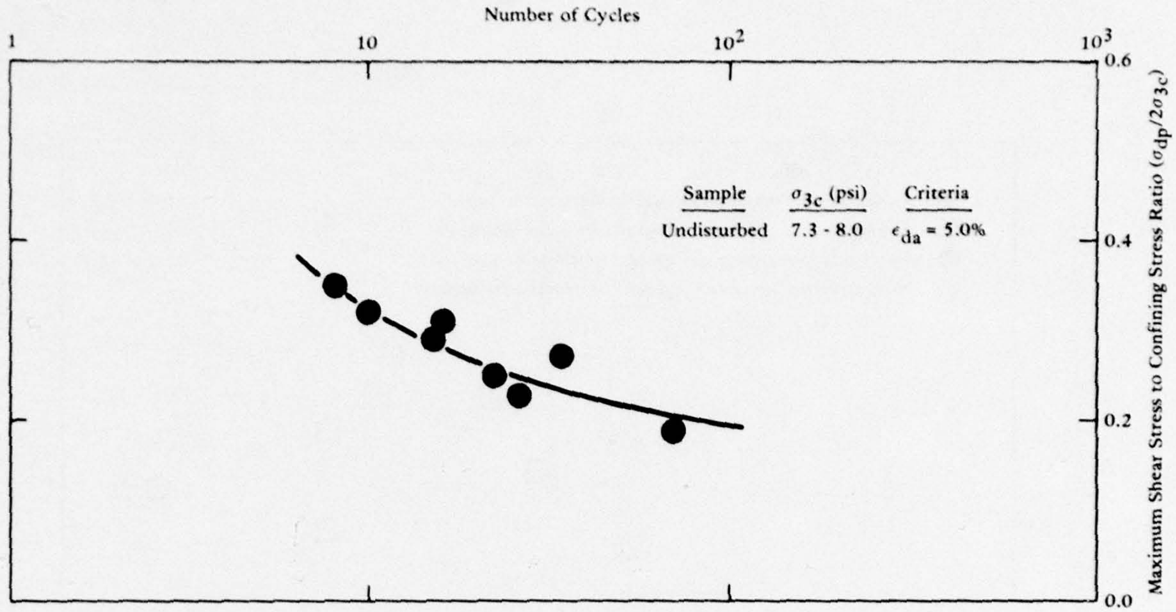


Figure 6. Liquefaction strength curve based upon vertical strain (Ref 3).

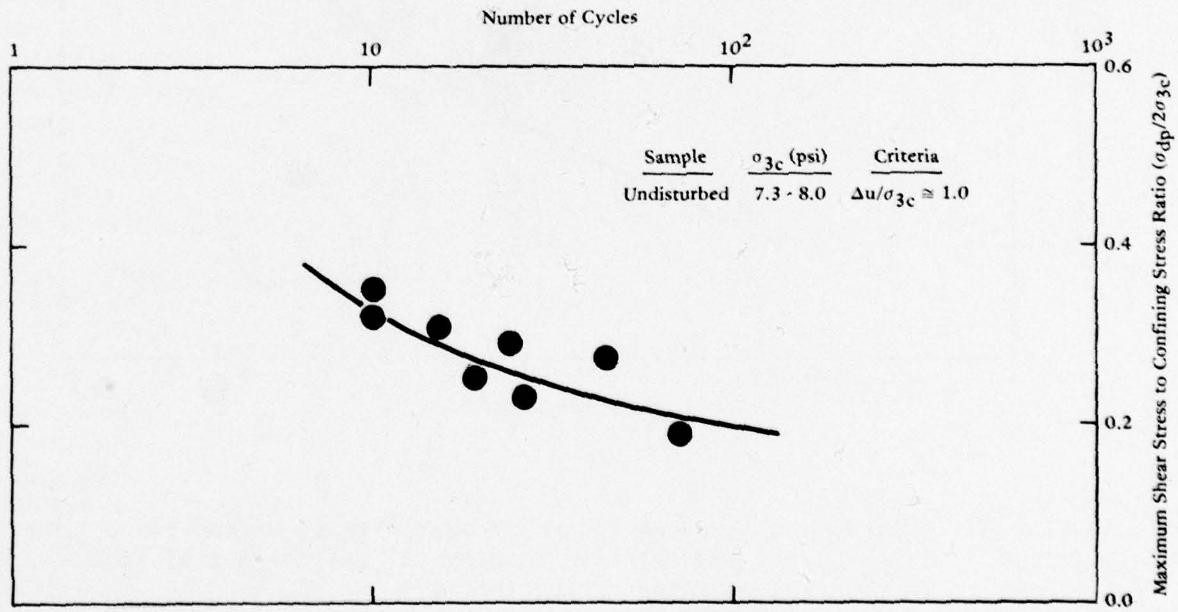


Figure 7. Liquefaction strength curve based upon pore pressure (Ref 13).

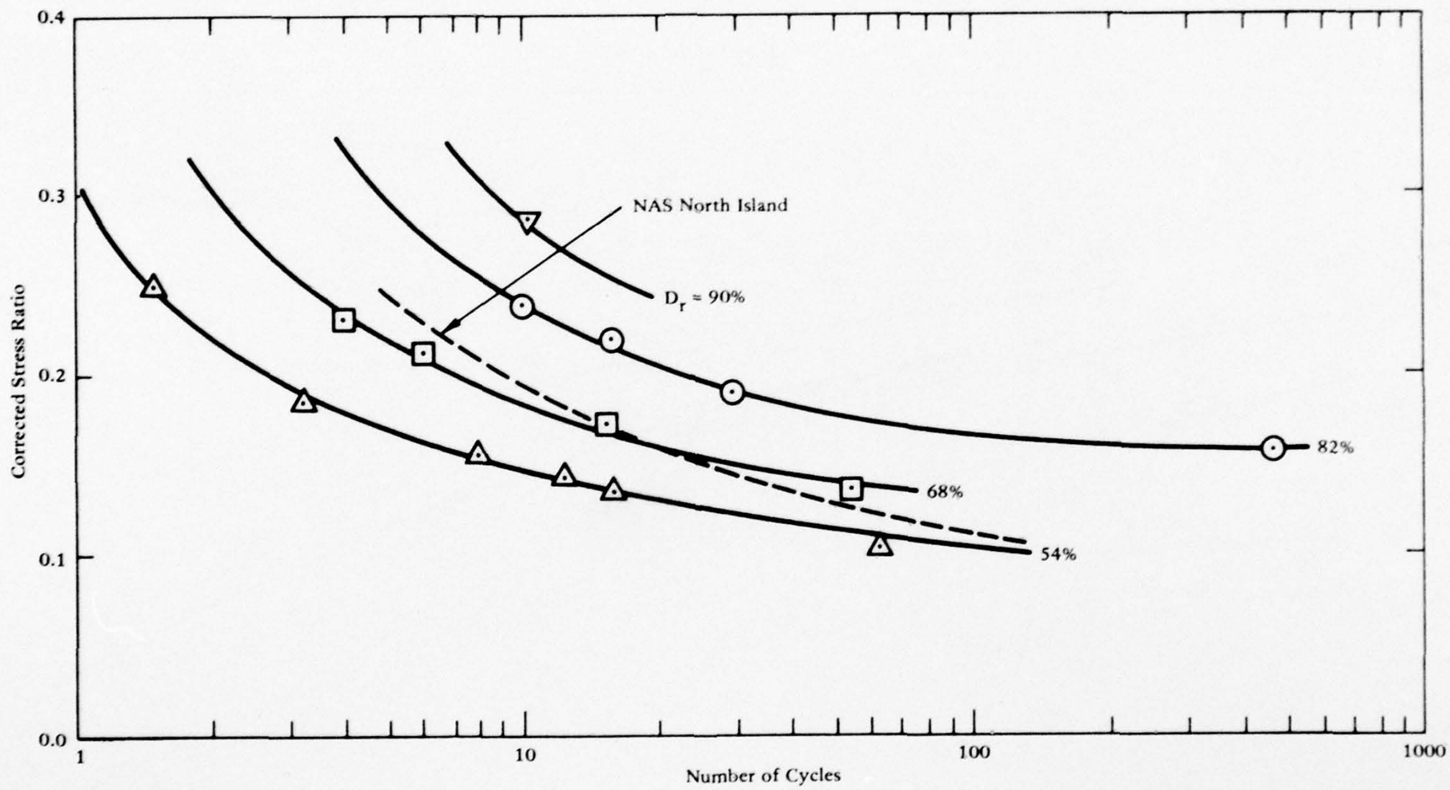


Figure 8. Shear ratio versus number of cycles for initial liquefaction.

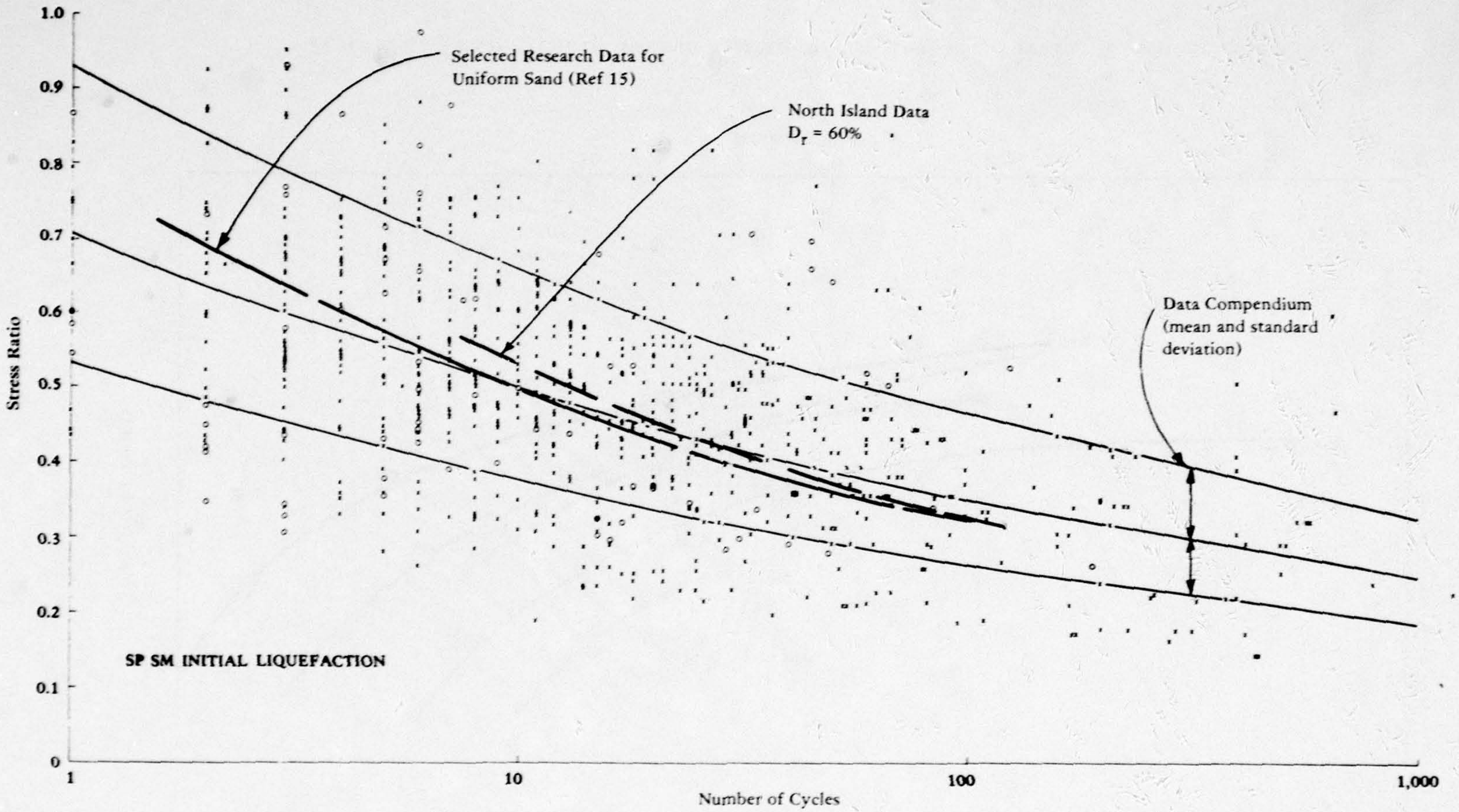


Figure 9. Normalized stress versus number of cycles to initial liquefaction (Ref 7).

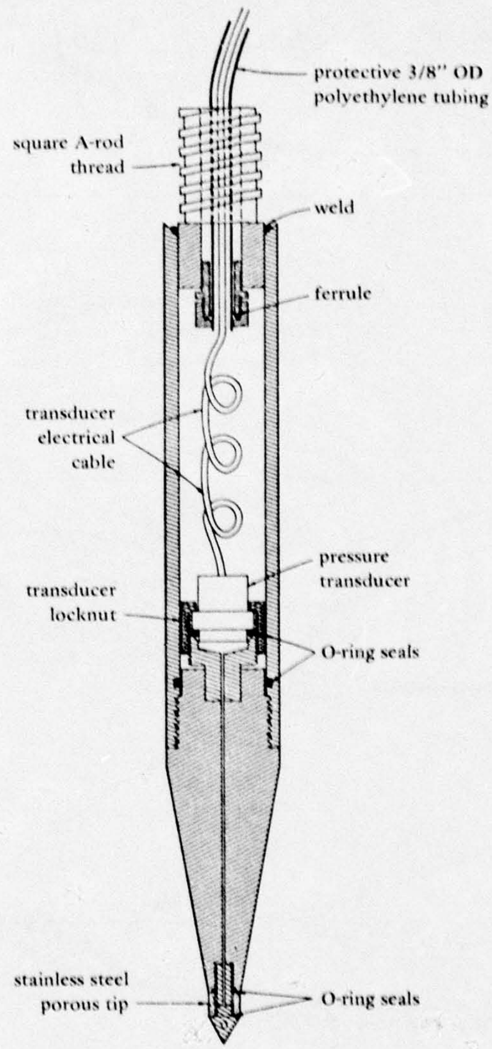


Figure 10. Schematic of the piezometer probe (Ref 16).

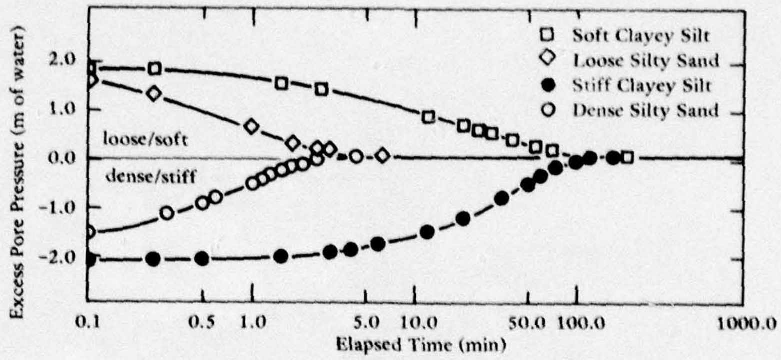


Figure 11. Pore pressures generated by the probe penetrating cohesive and cohesionless soil deposits (Ref 16).

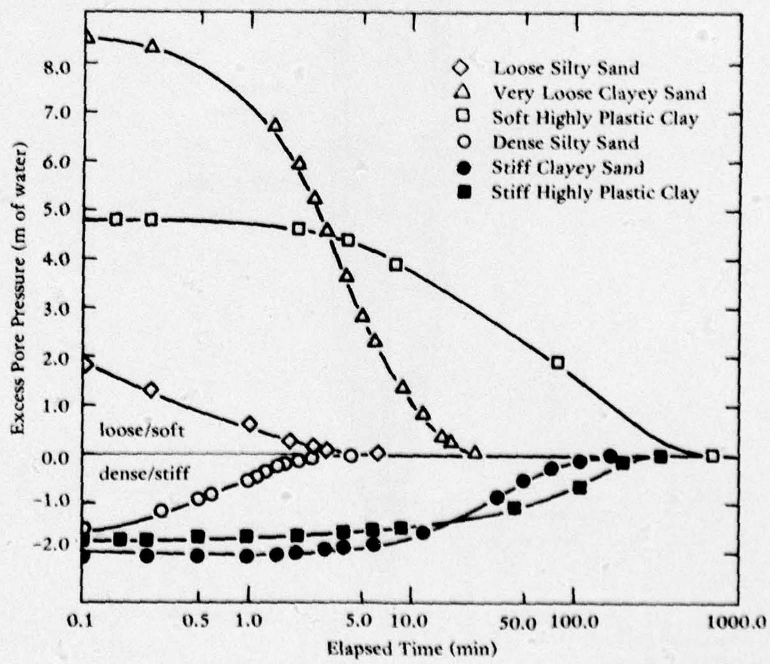


Figure 12. Time-rate of dissipation of excess pore pressure generated by the probe in dense/stiff and loose/soft soils (Ref 16).

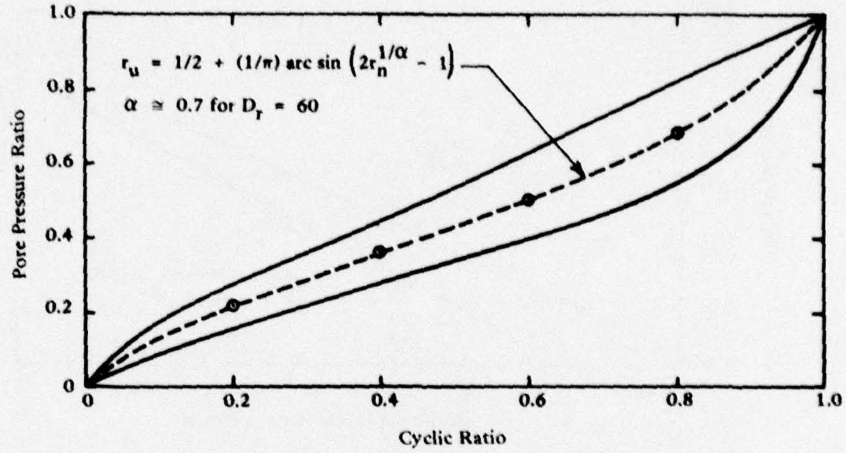


Figure 13. Rate of pore water pressure buildup in cyclic simple shear tests (Ref 10).

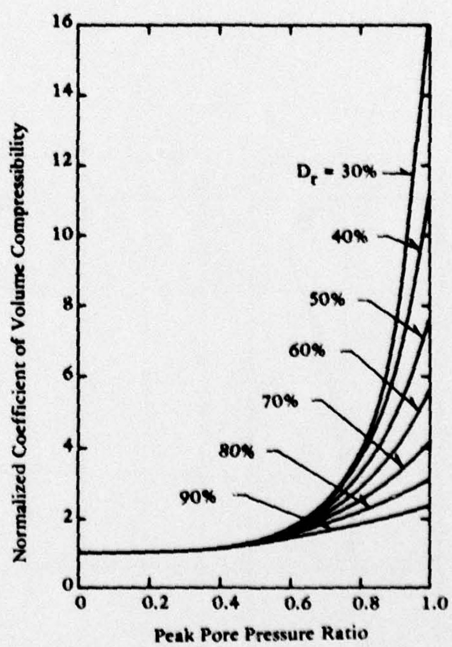


Figure 14. Theoretical relationships between compressibility of sands and pore pressure buildup (Ref 28).

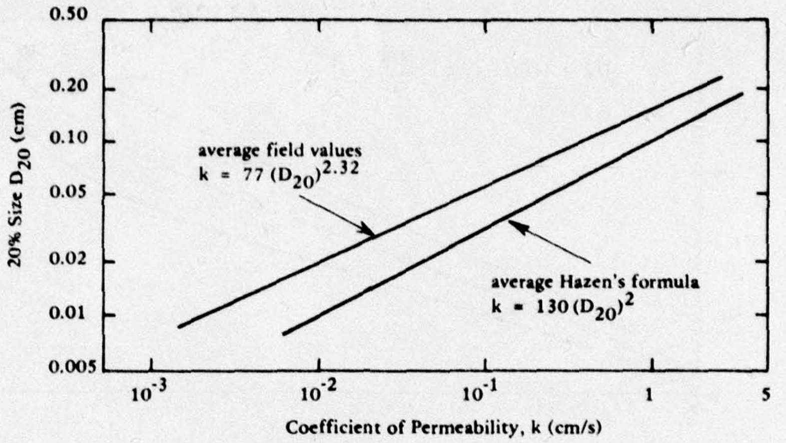


Figure 15. Relationships between grain size and coefficient of permeability for sands (Ref 28).

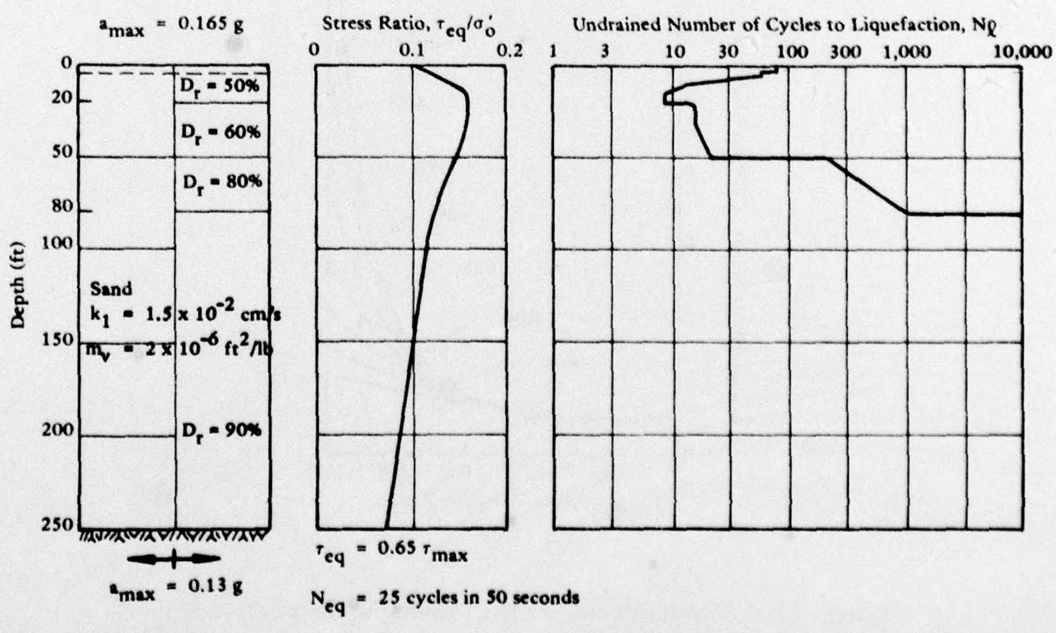
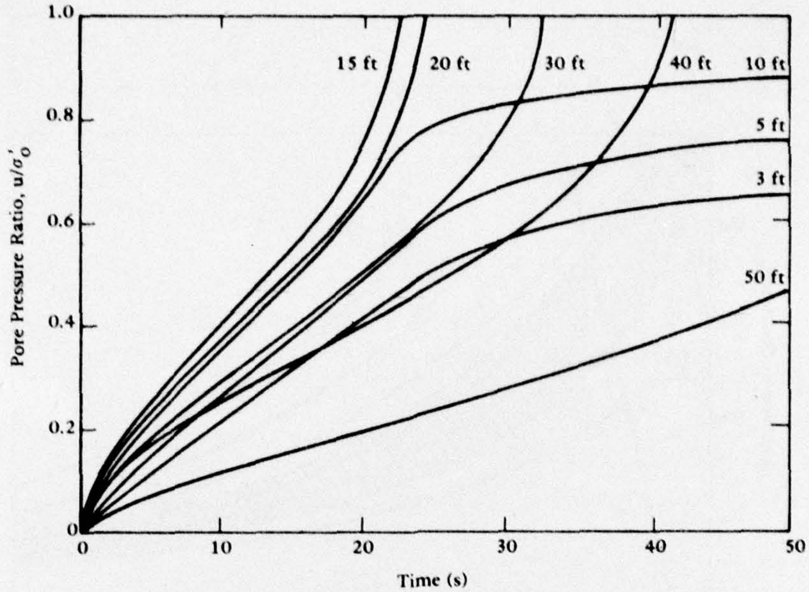
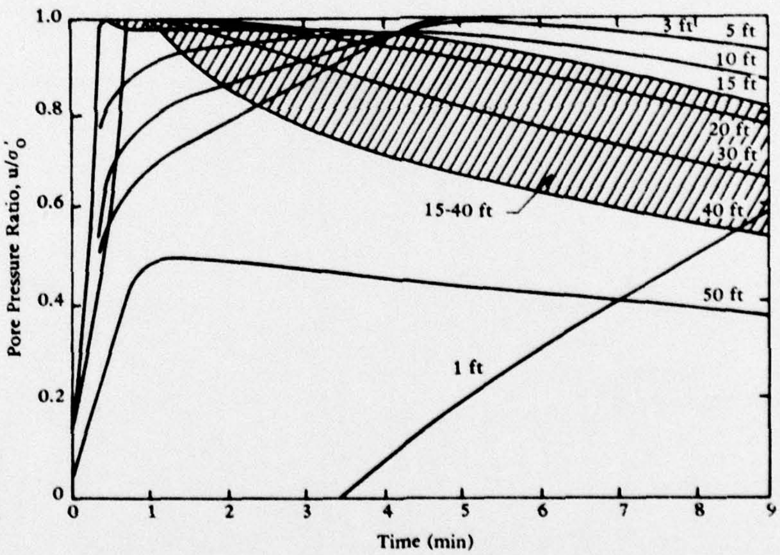


Figure 16. Soil profile and stress conditions used for analysis (Ref 28).

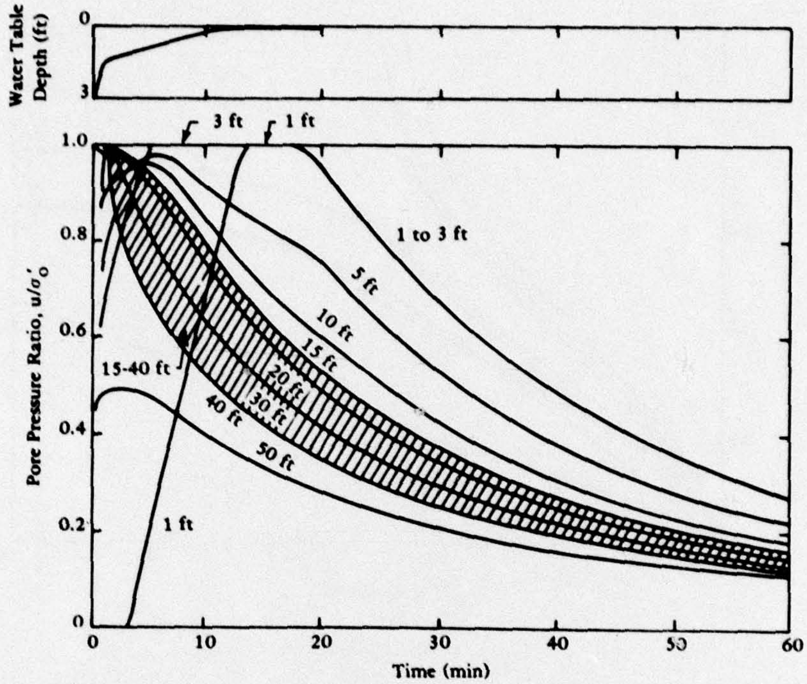


(a) During earthquake shaking.



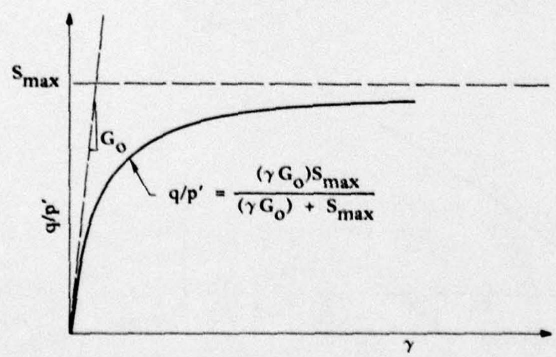
(b) In 8-minute period following earthquake.

Figure 17. Computed development and variation of pore water pressures for soil profile shown in Figure 16 (Ref 28).

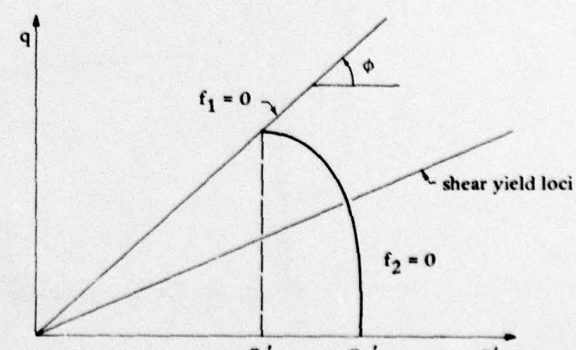


(c) In 60-minute period following earthquake.

Figure 17. Continued.



(a) Normalized stress-strain relationship.



(b) Stress path.

Figure 18. Ishihara material model.

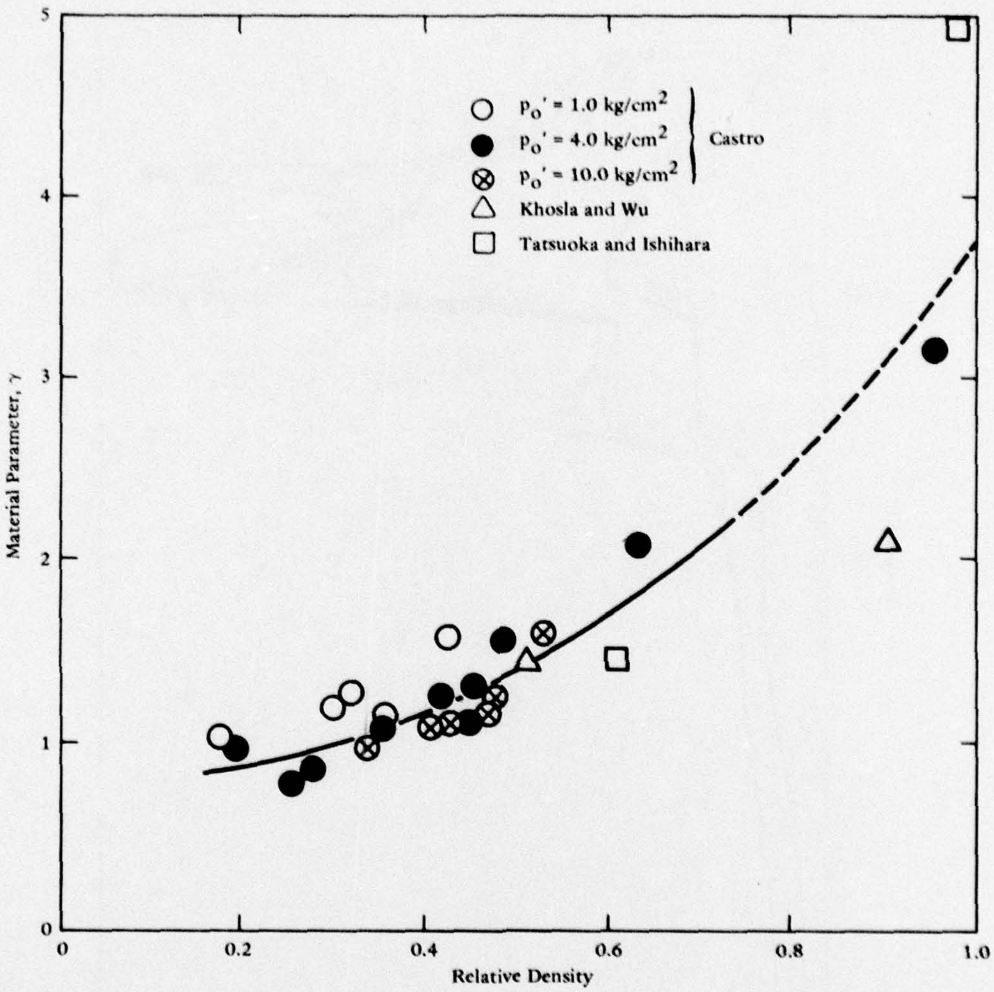


Figure 19. Relationship between material parameter λ and relative density.

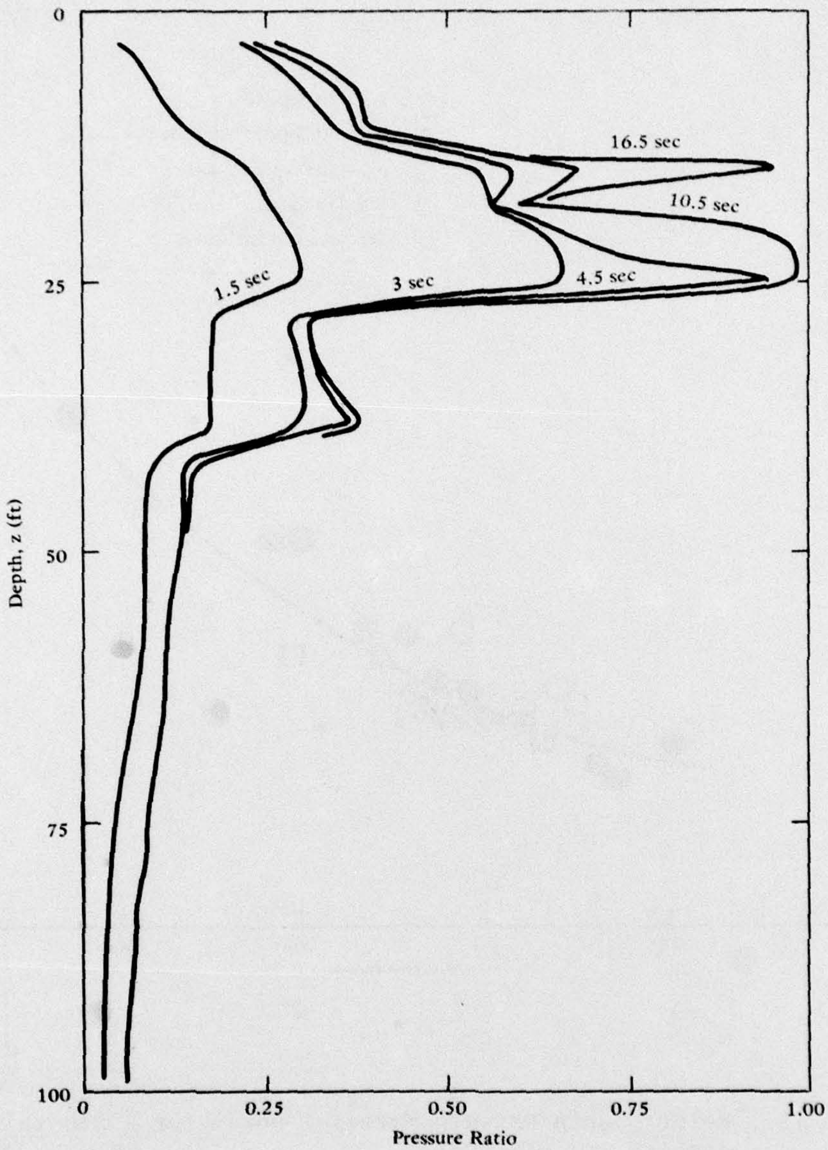


Figure 20. Variation of pore pressure with depth (Ref 32).

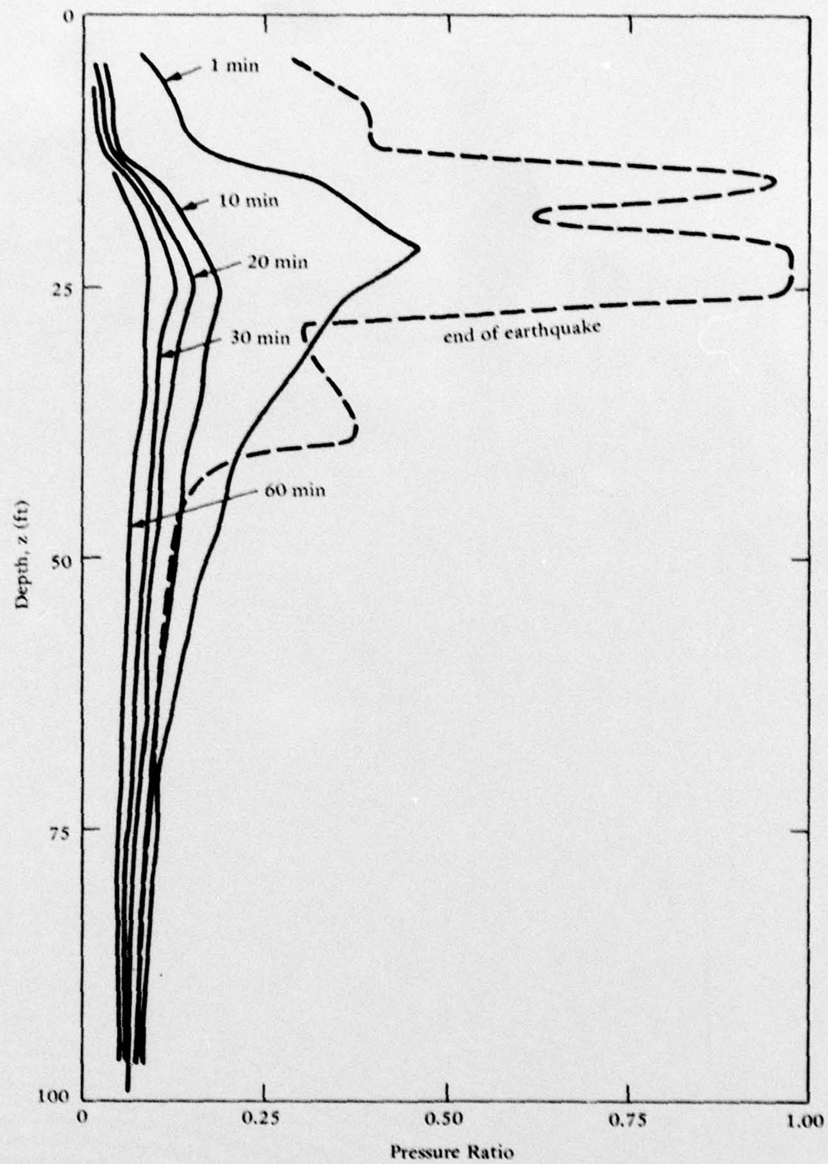


Figure 21. Post earthquake dissipation of excess pore pressures (Ref 32).

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LIST OF SYMBOLS

a	Acceleration
a_{max}	Peak acceleration
C_v	Coefficient of consolidation of the soil
D_r	Relative density of the soil
F	Formation factor, used to characterize a soil structure
f_1, f_2	Constants in soil model from Reference 32
G_o	Unloading shear modulus
H	Thickness of soil layer
k	Soil permeability
m_v	Soil coefficient of volume compressibility
N_ℓ	Number of cycles to liquefaction
n	Soil porosity, including non-interconnected voids
n_e	Effective soil porosity
n_{eq}	Equivalent number of stress cycles, used in conjunction with τ_{eq}
p'	Effective confining pressure
p'_f	Effective pressure at failure
p'_o	Initial effective pressure
q	Shear stress
S_{max}	Maximum possible shear stress to effective stress ratio
SM	Symbol for a silty sand
SP	Symbol for a poorly graded sand
u	Pore water pressure
x	Shape factor used to characterize particle shape

LIST OF SYMBOLS
(Continued)

z	Depth coordinate distance
γ	Symbol used to designate shear strain
γ_w	Density of water
Δu	Increment of pore water pressure
σ_{3c}	Lateral confining pressure
λ	Parameter used for soil model in Reference 32
$\sigma_{dp}/2\sigma_{3c}$	Ratio of maximum shear stress to confining pressure
σ'_o	Initial effective volumetric stress
τ_{eq}	Equivalent shear stress level
$(\tau_{hv})_{av}$	Average horizontal shear stress
$\partial u_g/\partial t$	Time derivative of rate of pore water pressure generation

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